CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

AUG 31 1960

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AUGUST 1960 **VOL. LV. NO. 8**

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Volume LV, No. 8. LONDON, AUGUST, 1960.

EDITORIAL NOTES

The Economics of Shell Construction.

IT has frequently been asked whether the numerous " shell " roofs of novel shapes now being erected in many parts of the world are adopted because they appeal to the eye of the architect or his client, because they are fashionable, or because they are the most economical solution of the problem of providing a roof. Some views on this question are to be found in the papers read at the International Colloquium on Construction Processes of Shell Structures held in September last in Madrid. Many of the authors of these papers (see page 326) strive to show that æsthetics are not the only reason for adopting thin-slab roofs of shapes which a few years ago would have been considered to be extraordinary and more fitted for structures at exhibitions rather than for industrial and commercial buildings.

The exceptional shape of the large roof of the Exhibition Palace in Paris was not adopted because of its economy but only because it was desired to have a spectacular structure—in this case architectural exhibitionism was all-important and cost was a secondary consideration. In many other cases, however, economy has been the primary reason for using a thin-slab structure. What are claimed to be the most extensive barrel-vaults with prestressed ties in the world are the roofs of a group of factories covering an area of nearly 800,000 sq. ft. in Egypt. The amount of concrete is about a cubic foot for each square foot of floor covered, which is about the same as for a number of north-light conoidal roofs in Poland; in an improved design of these structures (see this journal for January, 1958) the quantity was reduced to a third of a cubic foot.

The shuttering is an important factor in the cost of curved roofs. It is shown in one of the papers that travelling shutters are a little cheaper than ordinary fixed shuttering for barrel vaults if they are both used three or more times, but that if they are used four times the cost of travelling shutters may be only three-quarters of that of fixed shuttering. These prices are based on costs in Denmark, but if adjustment is made for the costs of labour and timber in Great Britain the result is much the same. Some forms of construction have been developed with the intention of avoiding the use of shuttering. It is claimed that one such form, comprising tubular metal frames covered with expanded metal and lightweight concrete, has the appearance and advantages of a reinforced concrete barrel-vault but costs one-tenth less if the same degrees of insulation and finish are provided.

Another roof erected without shuttering is for an assembly hall at a school in

North London, and is pentagonal in plan and covers 14,000 square feet. The construction comprises numerous suspended wires on which slabs of insulating material are laid. Mortar was applied pneumatically from below, and, although such mortar is several times more costly than an equal volume of ordinary concrete, it is said that the smaller volume and less weight result in lower total cost. The shape of the roof is almost a hyperbolic paraboloid for which timber shuttering would be expensive because of the small variation from the true hyperbolic-paraboloidal shape. The total cost is estimated to be 16*s.* 8*d.* per square foot, a third of which is the cost of the gunite and concrete cast in place; the cost of the insulation is 1*s.* 3*d.* per square foot. The total cost is almost identical with that, excluding the finishes, of a roof of a garage in Lincoln which comprises four contiguous square true hyperbolic-paraboloidal reinforced concrete slabs which were cast on wooden shuttering. Other designs for this roof had been considered, namely, a dome which would be costly as shuttering curved in two directions would be required over the entire area, and three cylindrical vaults of 96 ft. span which were thought to be less striking in appearance and more costly to erect. It is interesting to compare these costs of concrete roofs with the cost of similar timber roofs in Great Britain, particulars of which are given in one of the papers. A vaulted timber dome, 60 ft. square on plan, cost 14*s.* per square foot including finishes, and a hemispherical timber dome cost 17*s.* per square foot excluding finishes. The permanence of timber construction for a structure so exposed as a roof is questioned in one of the papers. Economy was sought in another case by not using a waterproof covering on a reinforced concrete roof over a car park. This is a hyperbolic-paraboloidal structure in the form of an inverted pyramid, and cracking is prevented by providing prestressed edge-beams the effect of which is to avoid or reduce tensile stresses in the slab. The cost of this roof, which is in Ceylon, was about 12*s.* per square foot. The economy resulting from using the shuttering for hyperbolic-paraboloidal roofs many times is emphasised in the case of the roof of the factory described on page 303.

The rectilinear generators of a true hyperbolic-paraboloidal structure favour precast construction and, as is shown in one of the papers, if the precast parts are prestressed during manufacture and after erection an economical roof is obtained so long as there are enough identical parts to offset the high cost of moulds strong enough to resist the tensioning forces. A notable example of shell construction embodying precasting is the barrel-vault roof of some hangars at Abingdon. This roof was cast in three parts, each of which is 186 ft. by 110 ft., weighs about 1400 tons, and comprises three barrel-vaults. These parts were cast separately on the ground below their permanent positions and jacked up at the rate of 1 ft. per hour to a height of 45 ft. The cost was one-tenth less than the estimated cost of the cheapest roof of similar shape with steel, aluminium, or prestressed concrete framing, and seventeen per cent. cheaper than the most costly of these forms.

In the case of corrugated, or " waved " roofs, in Germany it is stated that precasting trough-shaped parts saved more than three hours of construction time for each square yard of roof. More than a hundred north-light reinforced concrete shell roofs of 62 ft. span at Amiens were each constructed in four days with the use of travelling shuttering. " Shell " structures in general, because of the thinness of the slabs, require less materials than other forms of reinforced concrete construction, and, if there is also a saving of time, a " shell " structure may often be competitive in cost with other forms of construction.

The Design of Concrete Biological Shields for Nuclear Reactors.—I.

By S. GILL, B.Sc. (Tech.), A.M.I.C.E., A.M.I.Mech.E.

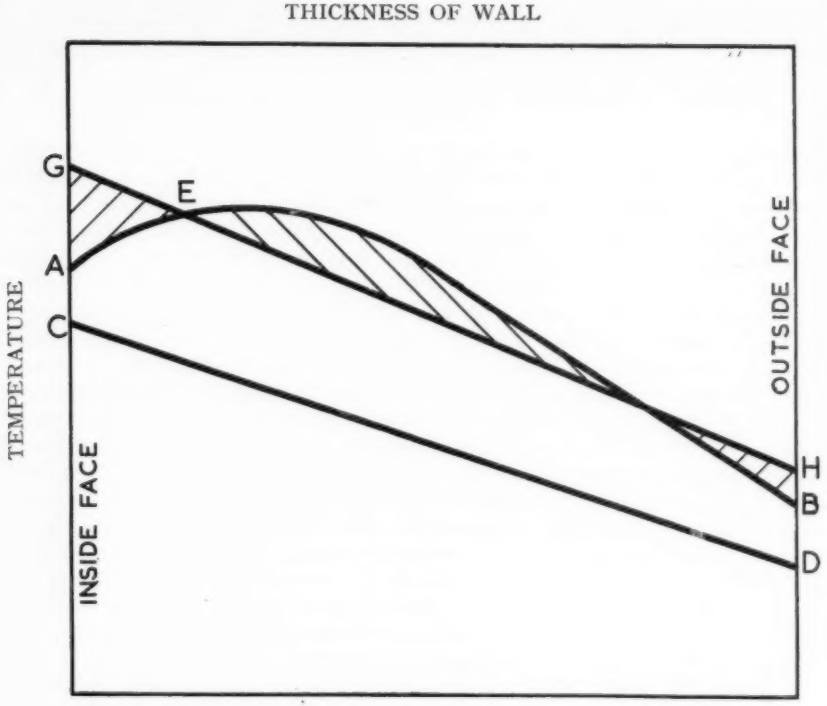
THE shape of concrete biological shields around nuclear reactors and the unusual conditions in which the shield operates are such that the methods of calculating thermal stresses in thin reinforced concrete members do not apply. The reactor structure comprises many redundant members, and the application of the principles of statics in its structural analysis is possible only when simplifying assumptions are made. The stresses due to the principal loads on the thick walls resulting from the superstructures and adjacent buildings are comparatively small. The thermal stresses are much larger than in most reinforced concrete structures, in which an appreciable error in the assessment of the thermal stresses is generally not significant as these stresses are usually small compared with the structural stresses; this relation is not necessarily true in the case of biological shields. To a large extent, the difference of temperature which a shield wall must sustain can be controlled by a system of air cooling acting on all inner faces, and in fact the thermal and structural stresses in the shield are maintained artificially at the same order of magnitude. The relative costs of reinforcement to resist thermal stress and of an extensive cooling system are considered in the design.

An important property of the concrete used in the construction of biological shields is the density under operational conditions, since the degree of nuclear shielding is, within limits, proportional to the density.

The density of the concrete forming the shield decreases with increase of temperature as moisture is evaporated. Concrete having a density of 150 lb. per cubic foot (2·40 g. per c.c.) at 110 deg. F. (43 deg. C.) may have a density of only 145 lb. per cubic foot (2·32 g. per c.c.) at 200 deg. F. (93 deg. C.). Accordingly, it is necessary from the point of view of density as well as of thermal stress to specify a limit to the temperature of the concrete. To require no more than an economical degree of cooling and to maintain a density of, say, 147 lb. per cubic foot (2·35 g. per c.c.) throughout the wall, a permissible working temperature of 150 deg. F. (65·5 deg. C.) in the concrete may be specified. During a temporary failure of power for the cooling system, it may be that a temperature of 212 deg. F. (100 deg. C.) can be tolerated.

The amount of heating due to nuclear effects may be small. If the cooling system is capable of removing most of the heat radiated from the pressure vessel and lagging, the temperature of the inner face of the concrete at any part of the wall can be adjusted in relation to the peak temperature inside the concrete. Moreover, by heating or cooling the spaces outside the inner shield, the difference of temperature through the wall can be minimized or even reversed. The adoption of this system of adjustment of temperature results in a gradual change of difference of surface temperatures up the wall, so that the temperature gradients vertically are not significant.

During the setting and hardening of the concrete in the biological shield, strains and relaxations of strain occur, and the resultant stresses in the concrete are indeterminate. These actions make it impossible for the present to forecast



AB.—Due to direct radiation of heat and nuclear heating combined.
CD.—Due to direct radiation of heat.
GH.—Equivalent rectilinear temperature gradient.

Fig. 1.—Temperature Gradients through Shield Wall.

rationally and accurately the stresses in the structure before the plant goes into operation, but research currently being carried out may lead to the quantitative evaluation of the effects of the actions separately, so that a comparatively accurate knowledge of conditions of stress immediately before the shield is subjected to operational heat may be obtained.

The heat generated by the setting of the concrete causes thermal expansions in the concrete. During construction and the subsequent period before operation, creep relieves some of the " locked-in " stresses, but the degree to which it does so decreases with time. The relief by creep cannot be computed confidently. Tensile stresses due to thermal contraction are imposed on the concrete when the rate of cooling becomes greater than the rate of generation of heat of hydration, and this process adds arithmetically to the stresses which simultaneously develop due to shrinking caused by loss of moisture. Superficial cracks relieve these effects in the concrete at the surface, but the internal stresses are only slowly, and incompletely, relieved by creep during the period before the reactor is in operation. During the process of stress-relieving the steel pressure

vessel within the area bounded by the shield walls, heat is likely to escape through the vessel and lagging, and cause further movements or stresses in the concrete, depending on the degree to which creep can operate.

Hence there are many variables affecting the computation of stresses, and, moreover, the properties of the concrete in relation to the flow of heat and the accommodation of strains may vary with time, temperature, and the degree of stress. These properties include thermal conductivity, specific heat, coefficient of thermal expansion, modulus of elasticity, tensile strength, and ultimate strain. These properties also vary according to the water-cement ratio, and the type of cement and aggregate. Most of the properties are therefore evaluated by laboratory tests with the particular cements, aggregates, and mixtures of concrete to be used.

During operation of the reactor, a difference of temperature consisting of two parts is set up in the shield wall. The first part is caused by the heat which escapes by radiation and convection from the pressure-vessel and lagging, giving rise to a linear temperature gradient through the wall by natural cooling. The second part results from heating caused by the absorption of neutrons within the shield. This effect is represented by a curvilinear gradient, roughly parabolic, with a peak near to the inner face. The two effects are considered together in design. Usually about 20 per cent. of the heat removed by the cooling-air system is caused by nuclear irradiation of the concrete, the remainder being caused by radiation from the pressure-vessel.

The usual method of design of biological shields adopts the well-known formula for a restraining moment M which is required to keep a member of thickness d plane under a difference of temperature θ deg., that is

$$M = \frac{EI\theta\alpha}{d} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which E is Young's modulus for concrete, I is the moment of inertia of the cracked section, and α is the coefficient of linear expansion.

The curvilinear temperature-gradient representing nuclear heating is superimposed on the linear gradient arising from the heat radiated direct from the pressure-vessel as shown in *Fig.* 1. The combined curve can be converted into an equivalent linear temperature-gradient by constructing a diagram in which the area under the equivalent line equals that under the curve, and for which the moments of each area about the temperature axis are equal. The difference of temperature given by the equivalent line gives the value of θ in formula (1).

Since the thickness of the shield and temperature-gradient may vary at different positions around the circumference of the shield and in a vertical plane, the shield is divided into parts. The conditions throughout each part are assumed to be uniform. The fixed-end bending moments for each part are then calculated and distributed in the usual way. The effects of the restraints from adjoining slabs are increased under conditions of differences of temperature. The deflections of the members meeting at the points of restraint can be equated by the normal methods of structural analysis, and the calculated moments added algebraically.

Other methods have been suggested for calculating stresses, on the assumption that the concrete cannot resist tensile forces. Mr. W. Bonsall[1] has suggested

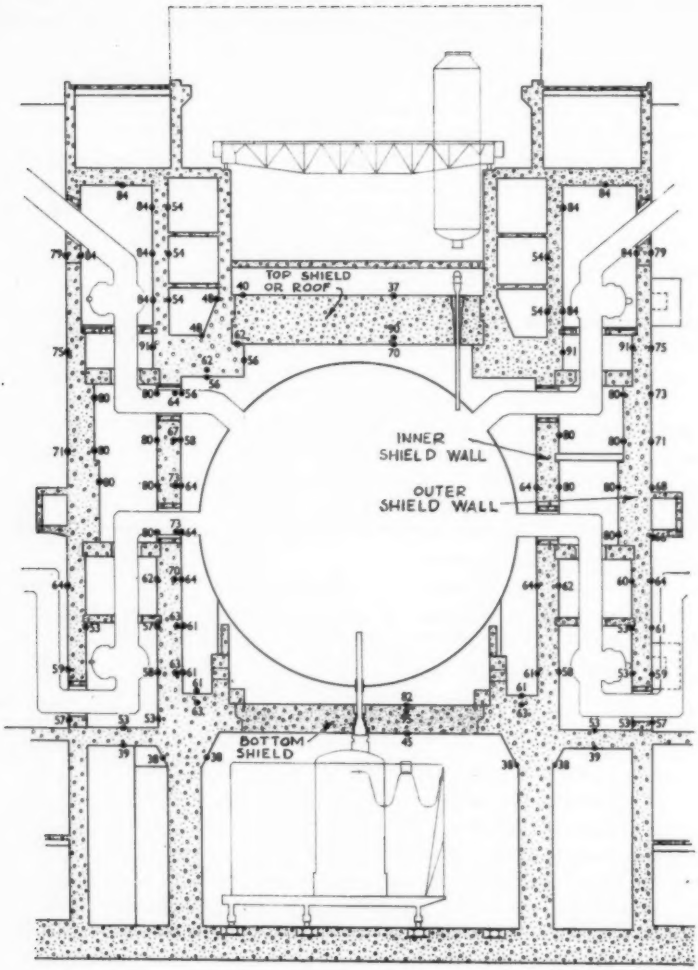


Fig. 2.

a mathematical method in which the wall is assumed to be restrained to remain planar but free to expand in a direction normal to its thickness. Temperature in the wall is considered to be a function of the distance from a face. The method cannot be applied directly to a shield in which non-uniform temperatures or thicknesses cause rotation of the supports. One result of this analysis is to show that, under certain conditions of restraint and distribution of temperature, only nominal amounts of reinforcing steel are required. Mr. Bonsall's method, if applied to the shield walls dealt with in the following example, gives similar results.

Example.

The analysis of the structural and temperature stresses adopted for the shield walls of the bottom-charge reactor in *Fig.* 2, while applying the principles described in the foregoing, makes important departures from previous practice. The structure comprises inner and outer concentric shield walls both approximately circular in plan. For the conditions obtaining in this shield, consideration of strain represents a more appropriate basis of design than stress, and there

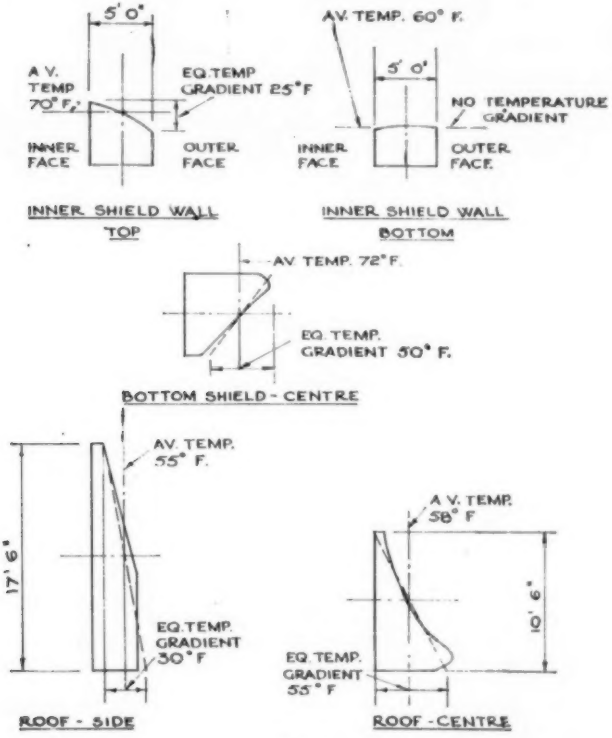


Fig. 3.

is no necessity to compute stresses under conditions of varying moments of inertia. The appraisal of restraint effects in a structure is related to the relative stiffness of its components. The difficulty of dealing with varying moments of inertia and movements of the neutral plane on cracking in structures such as thick-walled cylinders is apparent. It is therefore logical that the permissible limits of design should be based on the strain resulting from a tensile stress of 27,000 lb. per square inch in the reinforcement, since this stress cannot be exceeded without undesirable cracking in the surrounding concrete. Assuming Young's modulus for steel to be 30,000,000 lb. per square inch, the equivalent

strain is 0·0009. The critical strain assumed for purposes of calculation was 0·0005.

The greatest heating effects would occur, without cooling, at the equator of the shield and in the centre of the roof. The effect of the cooling system is to adjust the maximum temperatures and temperature-gradients at all positions around the shield to those shown in *Fig.* 2, from which it is seen that some parts of the wall above the equator are hotter on the outer face of the inner shield than on the inner face. Since the cross-sections considered for design do not in all cases coincide with those illustrated in the temperature-distribution diagram, interpolation is necessary to determine the temperatures used in the following calculations (*Fig.* 3). The temperatures shown in *Fig.* 2 are surface temperatures, whereas those used in the stress computations are derived from the equivalent gradients.

At the top of the wall the equivalent gradient is 25 deg. F., which results in circumferential stresses, tensile at the inner face and compressive at the outer face, of $\pm$150 lb. per square inch if the coefficient of linear expansion is assumed to be 0·000006 and Young's modulus of concrete is assumed to be 2,000,000 lb. per square inch.

Bibliographical References.

1.—Bonsall, W. "Thermal Stresses in Reinforced Concrete Shields." United Kingdom Atomic Energy Authority. Report No. 1, G.E. 1955.

Other bibliographical references to the design and construction of biological shields in concrete are as follows.—Ed.

Badland, P. A.—"The Structural Design of Reinforced Concrete Biological Shields." Reinforced Concrete Review, December 1957.

Gill, S.—"The Design and Construction of the Biological Shield at Hunterston." G.E.C. Atomic Energy Review, September 1959.

Bonsall, W.—"Stresses in Reinforced Concrete Shields for Nuclear Reactors." Proceedings of Inst. of Civil Engrs., July 1960.—In this paper the thermal stresses in a reinforced concrete slab with a distribution of temperature dependent only on the distance into the slab are analysed; shrinkage stresses, creep, and variation of physical properties with temperature are neglected. It is shown that in a slab which is constrained to stay flat a difference of temperature of about 50 deg. C. can be allowed with only a nominal amount of reinforcement to control cracking. On the other hand, if the temperature difference is greater than about 87 deg. C., then either the steel or the concrete must be stressed beyond the usual working limits. The economics of the alternatives of allowing a temperature difference of 87 deg. C. with the attendant necessity to provide heavy reinforcement or reducing the temperature difference to 50 deg. C. by means of a suitable "thermal shield" are examined; the thermal shield is probably the cheaper solution.

(*To be continued.*)

Hyperbolic-paraboloidal Roofs at Bedford.

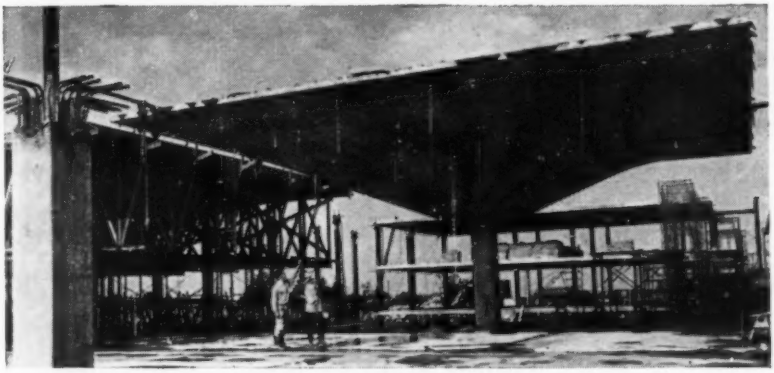


Fig. 1.

SOME of the roofs of the new factory for Texas Instruments, Ltd., at Bedford are thin hyperbolic-paraboloidal concrete slabs in the approximate form of a number of separate inverted pyramids (*Fig.* 1). There are twenty-nine of these units, each 48 ft. square, covering areas of about 408 ft. by 100 ft., 206 ft. by 100 ft., and 155 ft. by 100 ft. The columns supporting the roofs are at 51-ft. 6-in. centres in both directions, there being a gap 3 ft. 6 in. wide between each unit; the gaps are covered with lightweight roof slabs supported on steelwork which acts as a tie between the concrete units. The underside of the roof is 9 ft. above the first floor at the columns and 18 ft. at the edges of the concrete slabs.

The slabs are designed according to the membrane theory, and although a thickness of 2 in. would be sufficient as regards stresses, the slabs are 2½ in. thick to limit

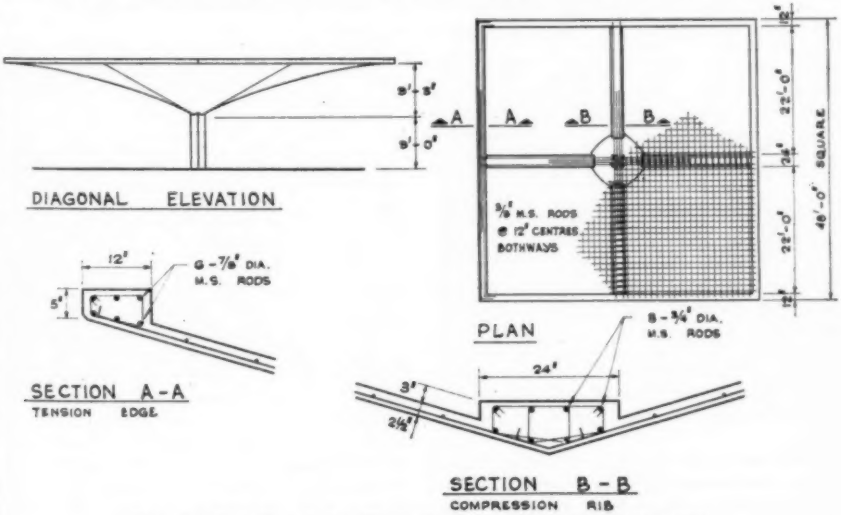


Fig. 2.—Details of the Hyperbolic-paraboloid Roof Slabs

Fig. 3.—Shuttering for a Hyperbolic-paraboloid Slab.

the deflection at the edges due to unbalanced loads. Four ribs extending from the head of the column to the edges (*Fig.* 2) are provided to resist compression and a shallow beam 1 ft. wide is provided along the edges to resist tensile forces. The slabs are designed to support a uniformly-distributed imposed load of 25 lb. per square foot, and a load of 100 lb. per foot along the edges to allow for the weight of ventilating ducts and the roofing covering the gaps. Allowance is made for unbalanced loads of 25 lb. per square foot on one half of each unit and 10 lb. per square foot on the other half, together with 100 lb. per foot on one half of the perimeter and 40 lb. per foot on the other.

The slab is thicker near the column and extra reinforcement is provided to resist the forces due to cantilever action. These forces are not readily calculable and were evaluated from the results of tests made in America and Great Britain. Neglecting to consider the cantilever action has led to cracking of similar structures. The deflection of the slabs was the subject of tests made by the Cement and Concrete Association on a model one-sixth of the full size. The model, which is illustrated in this journal for July 1960, was fixed in a frame and the base bolted rigidly to the floor. The load was applied by suspending sixty-four weights by cords from plates attached by plaster to the upper surface of the slab. The cords passed through holes in the slab. The weights were supported on a platform which could be lowered by means of two jacks, thereby applying the load gradually. The loads at the edges of the slab were applied by means of 5-lb. weights suspended from the edge. The deflections of the slab were measured by means of dial-gauges capable

of measuring 0·001 in., and the rotation of the column by an optical method. It was found that the edges of the slab might deflect about 2 in. under the unbalanced loads, about one-third of the deflection being due to the loads at the edges and two-thirds to the unbalanced distributed load.

An advantage of such roofs is that, although the slab is curved in two directions, the shuttering can be constructed entirely of straight boards. A part of the shuttering for one of the slabs is shown in *Fig.* 3. The wooden shutters are supported on a horizontal square steel frame. Screw-jacks at the corners of the frame are used to raise and lower the shuttering. Three sets of shutters were constructed and moved from one unit to the next on rollers under the base-plates of the jacks. The shuttering was moved three days after the unit had been concreted.

The concrete for the roof was mixed in the proportions of 4·8 to 1, and had a slump of 2 in. which ensured that the concrete was stiff enough to be placed on the sloping shuttering without slipping. The aggregate was of ¾ in. maximum size. The thickness of the slab was regulated by short screeds and the surface was trowelled smooth to receive sheets of bituminised felt.

The foundation of each unit is an independent footing bearing on boulder clay with a maximum intensity of pressure of 4 tons per square foot.

The architects are Messrs. O'Neil Ford & Richard Colley of the U.S.A. The consulting engineers are Messrs. Oscar Faber & Partners and the general contractors are Tarmac Civil Engineering, Ltd. The cost of the structural roof slabs, ribs, edge beams and columns is estimated to be about 5*s.* per square foot.

The Quality of "British Standard" Products.

A CORRESPONDENT writes :

"It is probably some twenty-five years ago that you first commented on the low quality of some products made in accordance with the requirements of British Standards. Since then your comments on the shortcomings of many new and revised British Standards for materials and products used in the concrete and allied industries have been useful and instructive. During all these years 'Concrete and Constructional Engineering' and 'Concrete Building and Concrete Products' have been almost alone in studying and criticising British Standards. In one of your numbers you expressed the view that some of these British Standards were standards of inferiority, instead of high quality as one might expect from their title.

"It seems that at last your views are accepted in high places, for in the House of Commons recently, in reply to a suggestion that there should be 'greater recognition of the Kite mark of the British Standards Institution', the Parliamentary Secretary of the Board of Trade said that this Kite mark was a safeguard against shoddy goods, but some people thought that it was the lowest common denominator and not a standard of excellence.

"As you pointed out a few years ago, the British Standards Institution accepts no responsibility for the quality of the goods on which its Kite mark appears. This mark means only that the Institution has tested some samples of the products of the maker who has applied for permission to use the Kite mark and has found that they comply with the relevant British Standard, which was prepared by a committee composed mainly of representatives of firms making such goods. This can be of small service to the consumer, who may be excused if he expects goods bearing the mark of a body with such a high-sounding title to be of the very best standard of excellence."

Book Reviews.

"Reinforced Concrete Reservoirs and Tanks." By W. S. Gray. Fourth edition; revised by G. P. Manning. [London: Concrete Publications, Ltd. 1960. Price 12*s*.]

FOR nearly thirty years, this book and the complementary volume on water-towers have been and remain the only text books published in Great Britain dealing solely with water-containing structures. The text has been brought up to date and several new examples of the design and construction of such structures are included. The appearance of a new edition at the present time is fortuitous in view of the publication recently of the new British Standard Code of Practice No. 2007 (1960) dealing with the design and construction of reinforced concrete and prestressed concrete structures for the storage of liquids. The requirements of the Code as regards reinforced concrete structures are dealt with by means of charts and examples. Prestressed concrete structures are not dealt with nor are lined tanks to contain corrosive liquids nor tanks containing hot liquids. As in the previous edition, the complete analysis of Dr. Reissner's theory, by means of which the bending moment on the walls of cylindrical tanks can be calculated, is given together with charts to enable the method to be readily applied in practice. The new matter includes statically-indeterminate structures analysed by the displacement method, and an analysis of the forces to which gas-holder tanks are subjected. As before, working drawings and illustrations of work in progress are a feature of this publication.

"Concrete in Building Work: Materials and Techniques." By W. G. Gregory. (London: Oxford University Press. 1960. Price 7*s*. 6*d*.)

THIS book, which is published by Hong Kong University, contains about forty pages of text and about the same number of blank pages. The materials for, and the making of, ordinary and special concretes, reinforcement, and decorative finishes are dealt with by means of short "jottings" in a form such as a lecturer might use. This form has the advantage of being concise so long as the notes are factual, as most are; some could be expanded. For example, indefinite statements such as "the amount of shrinkage is also dependent on this [water-cement] ratio" could be made more informative if it was explained briefly that the greater the ratio the greater the shrinkage. Most of the data is abstracted from British standards. The special requirements in Hong Kong regarding water are interesting. The notes are addressed to architects and architectural students; engineering aspects of the subject are specifically omitted.

"Foundation Design and Practice." By J. H. Thornley. (London: Oxford University Press. 1960. Price £6.)

THIS book of three hundred large pages is remarkable for two reasons; it approaches the design of foundations from an economic viewpoint which considers conditions in the U.S.A., and there is a complete absence of formulæ. An "elimination table" is given from which an engineer can select the most suitable type of foundation for wide ranges of condition of ground, magnitude of load, and type of structure. Most types of foundations are considered and many types of piles in use in America are described; prestressed concrete piles are not included. The book is written in an almost chatty style and the index is much more comprehensive than is usual.

"Sell's Building and Civil Engineering Trades List, 1960." (London: Business Dictionaries Ltd. Price 40*s*.)

THIS annual publication has been retitled to emphasise its use to the civil engineer as well as the builder and the format has been altered. The directory now contains eight sections, namely new products, builders and contractors, builders' merchants, concrete, contractors' plant, floor ing, plastics, and trade marks.

"L.M.B.A. Handbook 1960." (London: London Master Builders' Association. Price 7*s*. 6*d*.)

THE current edition of this Handbook has been completely revised in content and make-up and contains 480 pages. The sections include membership, labour matters, contract matters, public authorities, technical data, publications, and memoranda for builders and contractors. New matter includes lodging allowances. The section dealing with education and apprenticeship has been rewritten.

The Design of the Elevated Road at Hammersmith.

THE elevated road, which is to be constructed at Hammersmith, London, and which was illustrated in this journal for November 1959, is about 2000 ft. long and has two 24-ft. carriageways which carry two lanes of traffic in each direction between the Great West Road and the extension of Cromwell Road. Some notes on the design are given in the following.

The superstructure comprises precast members assembled as shown in *Fig.* 1 and prestressed together longitudinally. It will be supported by fifteen hollow tapered columns cast in place at intervals of about 140 ft. and will provide a clearance of about 16 ft. 6 in. over the roads and car park beneath. The approach roads are on filling between cast-insitu reinforced concrete retaining walls.

Prestressing.

The superstructure is to be prestressed longitudinally by means of 1¼-in. wire-ropes which will be deflected and will be kept in position by steel saddles in the cavities in the hollow precast units forming the central longitudinal beam through which the wire-ropes will pass. The wire-ropes will be anchored in the upper slab of the beam, and they are to be

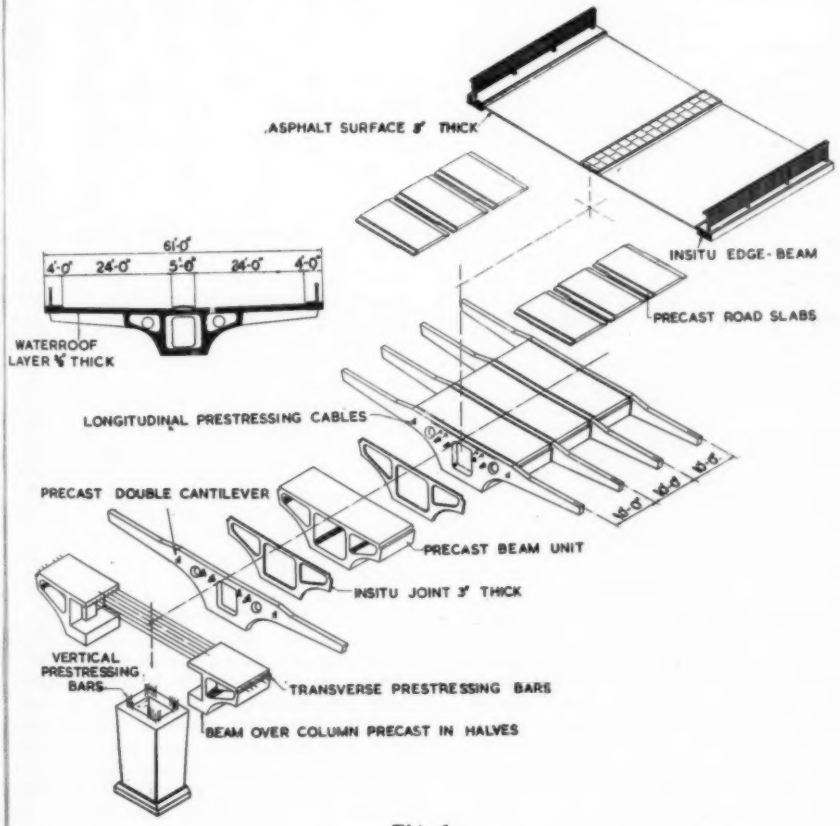


Fig. 1.

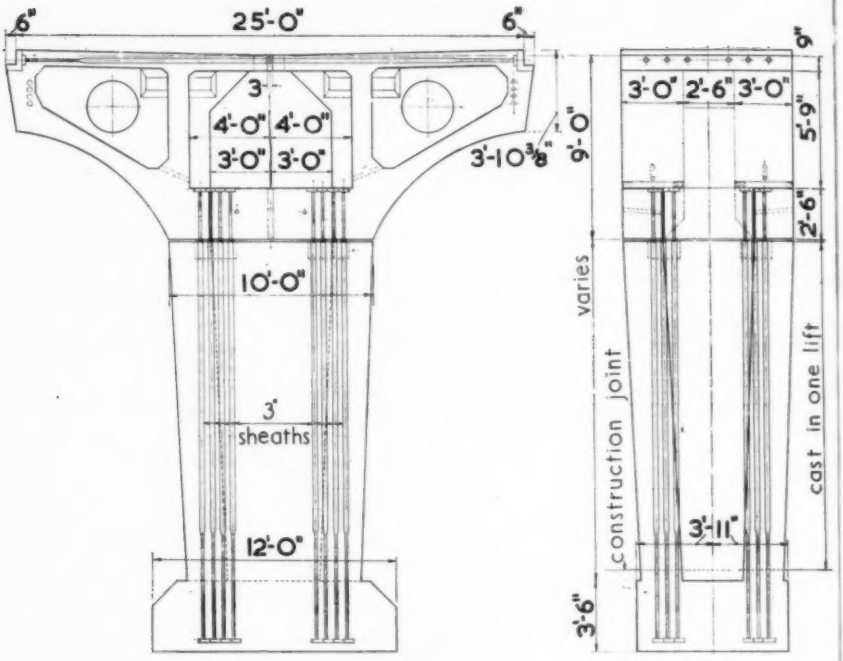


Fig. 2.

rectilinear between the saddles. Transverse and vertical prestressing at the columns will be by means of 1¼-in. high-tensile steel bars.

Each double cantilever and each block forming the longitudinal beam will be precast in one piece. It was intended that the part of the beam over each column should be cast in two pieces as shown in *Fig.* 1. It is likely, however, that each of these parts will be cast as one block which will be rigidly connected to the columns by the vertical prestressing bars.

Bearings.

Roller bearings at the bottom of each column will enable the top of the column to move as the superstructure expands or contracts due to change of temperature. This movement, which is expected to be about 6 in., will be taken up at a comb-type expansion joint near the middle of the structure. Details of the bearings at the bottoms of the columns are given in *Figs.* 3 and 4. The two small rollers (a), which are to be provided under each column, are to be of a German type and are to be made of special steel subjected to a process of deep case-hardening. As a measure of safety, four stools (b) are provided to support the column should any part of the bearing fail. During construction and before the bearings take the load, the columns will be supported by jacks on temporary bearing plates (e). Differential settlement of the foundations in excess of ½ in. will be compensated for by jacking up the columns at the mild-steel bearing plates (f) and inserting or removing shims (c) from beneath the roller bearings.

Components.

The hollow columns will be of rectangular cross-section. Each is to be prestressed by means of twenty-four 1¼-in. bars, which will be screwed into 2-in. anchor plates embedded in the bottom of the column (*Figs.* 3 and 4) and will be contained in sheaths of 3 in. diameter in the walls of the columns. Recesses will be formed in the tops of the columns to contain cylindrical couplings which are

screwed on to the upper ends of the bars. When the beam block is in position at the top of a column and the mortar in the 1-in. joint has hardened, short lengths of bar will be screwed into the couplings through the bottom slab of the beam and the bars will be tensioned and anchored inside the beam. The precast reinforced concrete double cantilevers are to be at 10 ft. centres and will be placed between the beam units. A concrete joint 3 in. thick is to be cast in place between the precast members. The deck is formed of precast reinforced concrete slabs 8 in. thick. When the slabs are in place the deck will be made monolithic by reinforcement embedded in cast-insitu concrete joints at the cantilevers. A cast-insitu edge beam is to be provided to distribute the load on to the cantilevers.

The safety-rail which is to be fixed to the edge beam is of a type developed in Sweden and is said to have stopped a loaded vehicle weighing 15 tons and travelling at about 30 m.p.h. at right angles and at 45 deg. to the rail.

The superstructure is to be anchored to the reinforced concrete abutments by prestressing bars which are designed to resist one-and-a-half times the greatest longitudinal force exerted by the superstructure.

Loads.

The structure has been designed generally for the British Standard load type H.A. To determine the greatest bending moments and shearing force the load is assumed to occupy four lanes simultaneously. To determine the maximum twisting moment in combination with bending the load is assumed to occupy the two lanes of one carriageway only. In addition the slabs and cantilevers in each lane of both carriageways are designed to support a load of 1 ton per linear foot on a length of 80 ft. simultaneously with a concentrated load of 6 tons from each of four wheels spaced in line across the loaded carriageway, two in each lane. A force of up to 25 tons is considered to act longitudinally due to tractive effects. Winds of velocities up to 70 m.p.h. are considered to act laterally on the net exposed area, an allowance of 8 ft. height above the carriageway being made for vehicles. The greatest wind force longitudinally was assumed to be half the greatest wind force laterally. An upward pressure due to wind of 5 lb. per square foot on the plan area of the structure is also assumed.

A coefficient of friction of 0·03 is assumed for the roller bearings, and is the coefficient commonly used in Germany for this type of bearing.

The foundations of the columns are founded on ballast overlying London clay. Investigation of the site indicated that differential settlement of the foundations might be small, but the structure will be capable of withstanding the stresses due to a differential settlement of 1 in. under the worst conditions. It is intended to limit the effective settlement to $\frac{1}{2}$ in. by jacking up the columns and raising the bearings. The greatest load on a bearing is about 1300 tons due to dead load and 420 tons due to live load.

Stresses.

The crushing strength at twenty-eight days of the concrete is to be at least 6500 lb. per square inch for the precast beams, cantilevers, and cast-insitu columns and 4000 lb. per square inch for the abutments, retaining walls, deck slabs, and edge beams, the aggregate being up to $\frac{3}{4}$ in. in size. The concrete for the joints between the precast units is to have a strength of at least 7500 lb. per square inch at twenty-eight days, the aggregate being up to $\frac{3}{4}$ in. in size. The prestressing wire-ropes will be encased in concrete having aggregate up to $\frac{3}{4}$ in. in size and a strength at twenty-eight days of at least 4000 lb. per square inch. Concrete of less strength will be used for other parts of the works.

The maximum permissible compressive stress due to prestress and longitudinal bending in the beams is assumed to be one-third of the crushing strengths of the concrete in the precast beams and joints, that is 2150 lb. and 2500 lb. per square inch respectively. No tensile stress due to bending is allowed and the greatest permissible principal tensile stress due to prestress, bending, shearing, and twisting is 150 lb. per square inch.

An initial force of 65 tons, which is about 80 per cent. of the tensile strength, is to be applied to the longitudinal wire-ropes and a reduction of 10 per cent. is expected due to relaxation of the steel. An additional reduction of about 17 per cent. is expected due to friction, creep of the concrete, slipping of anchors, and

successive stressing of separate wire-ropes. Measurement of the stresses is to be made during the erection to determine the behaviour of the wire-ropes. Should the force in a wire-rope be less than the required final force of 116,000 lb., the wire-rope will be re-stressed.

The net increase of pressure on the ground under the foundations of the columns, abutments, and retaining walls, is assumed to be 2 tons per square foot due to dead load only, and 4 tons per square foot under the worst conditions of live and dead load.

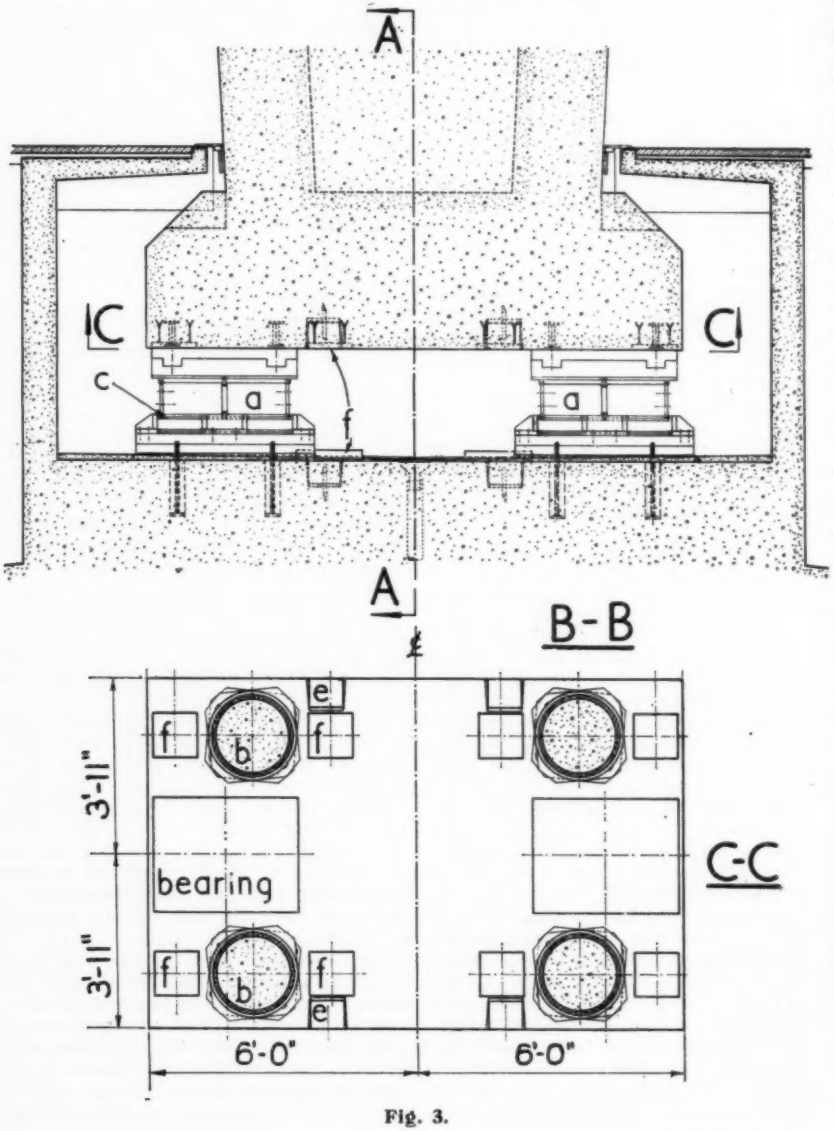


Fig. 3.

Design.

The longitudinal bending, shearing, and twisting is assumed to be resisted only by the central longitudinal beam, no account being taken of the deck or edge beams. To calculate the greatest twisting moment on the superstructure it is assumed that each beam is fixed at each end by the columns. This condition is more adverse than can actually occur since the deflection of the columns will distribute the torsional effects to the adjacent spans. The torsional shearing stresses in the member are assumed to be distributed in accordance with the membrane-analogy theory. The greatest shearing stress due to twisting is calculated to be about 200 lb. per square inch and is combined with the direct shearing stress near the columns; the prestress required is checked to ensure that the principal tensile stress due to bending and combined shearing is less than 150 lb. per square inch. The cantilevers are assumed to be fixed at the central beam and to be partially supported by the edge beams which are assumed to act as beams on elastic supports distributing the load on one cantilever to adjacent cantilevers.

Since there is no satisfactory theoretical method of analysing the stresses in the central beam, photo-elastic investigations were carried out at Southampton University to determine the amount of reinforcement required to resist the concentrated tensile stresses at the corners of the upper slab of the beam units. The analysis is made on the assumption that the structure will behave elastically under working loads. A prestressed concrete model one-twelfth full size was constructed comprising four beam units and four cantilevers on both sides of a column and was tested by the Cement and Concrete Association

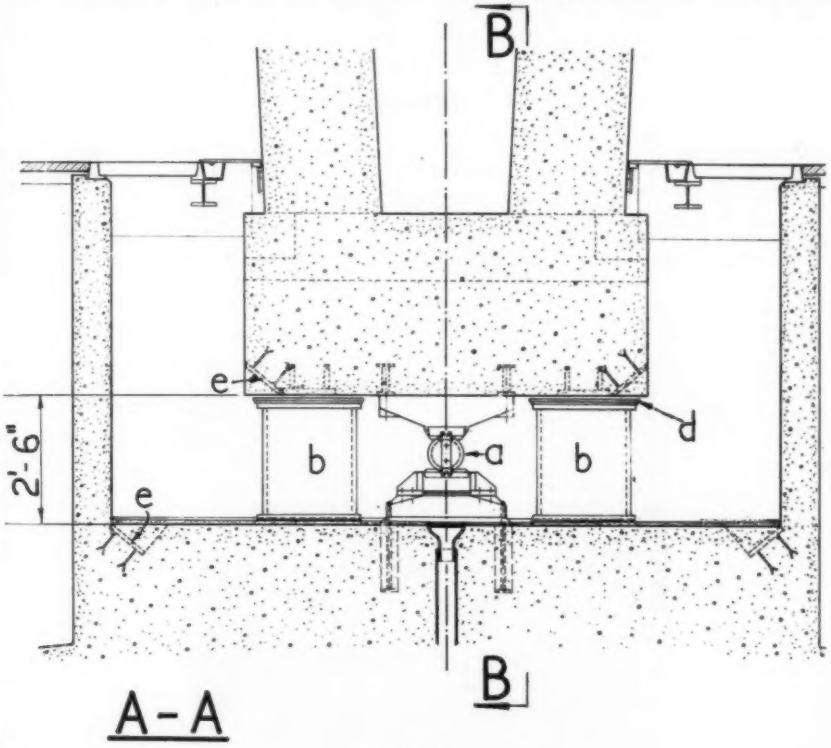


Fig. 4.

to determine the behaviour at conditions approaching failure. Two cases were considered, the first being a load-factor of 1·5 applied to the dead load, 2·5 to the live load on two lanes of one carriageway, and 1·0 to the live load on the other carriageway. The model carried this loading although extensive cracks were formed. The model was then unloaded and reloaded until it failed under the second mode of loading in which load-factors of 1·5 applied to the dead load, 2·5 to the live load on one carriageway and no load was imposed on the other carriageway. The model collapsed under conditions very near to this case, indicating that the structure should be sufficiently strong.

The consulting engineers are Messrs. G. Maunsell & Partners, in association with Mr. J. Rawlinson, C.B.E., M.Eng., M.I.C.E., Chief Engineer of the London County Council, and Mr. H. Bennett, F.R.I.B.A., Architect of the London County Council. The contractors are Messrs. Marples, Ridgway & Partners, Ltd.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", August, 1910.

Precast Construction of a Sea-wall.

"The new system of sea walling now being carried out by the Sydney Harbour Trust, at Miller's Point, is the first time the trestle wall system has been adopted. After the bottom had been prepared and levelled, the trestles, weighing 9 tons, were swung into place by a floating 15-ton crane. Including picking up and bringing them to the site and lowering them into place, eight trestles could be set up in one day, or 28 ft. of wall. For lifting, wooden toggles were put through the holes in the webs and wire rope slings slipped over, so that when lowered into the water the trestle assumed a truly vertical position. The centre of gravity in air was not quite the same as in the water, the trestles not being wholly immersed at any time. The correct position of the slings was found after a few trials."

The New Bridge over the River Tay at Perth.

THE original Victoria Bridge over the River Tay at Perth, Scotland, was a steel structure of four spans supported on masonry piers, and is being replaced by a prestressed concrete bridge of three spans (*Fig.* 2). The new structure, which is 400 ft. long, is on the same site as the old bridge and is of cellular construction comprising four prestressed longitudinal girders, two reinforced concrete spandrel girders, a cast-insitu slab extending across the soffits of the six girders, and

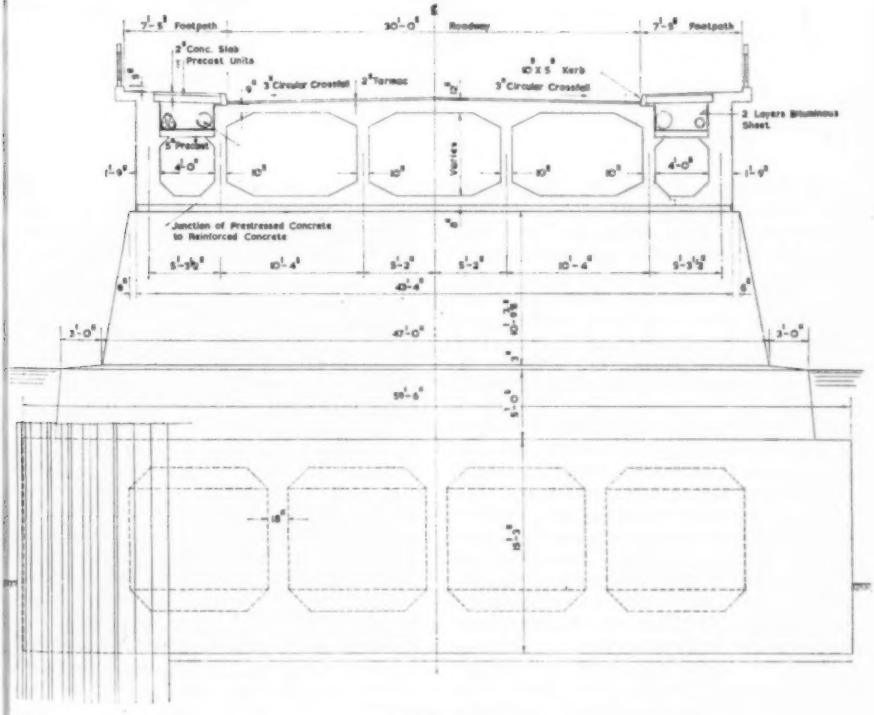
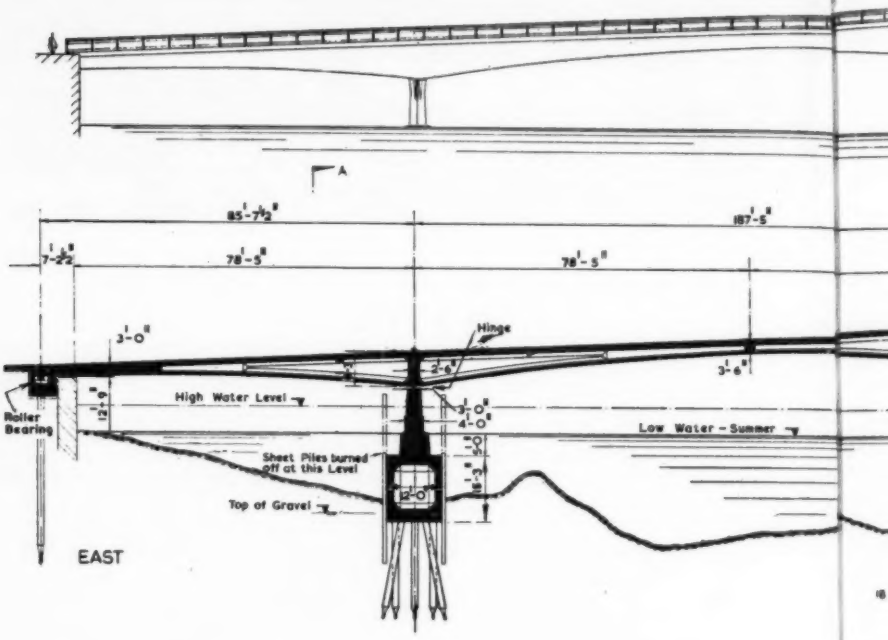


Fig. 1.



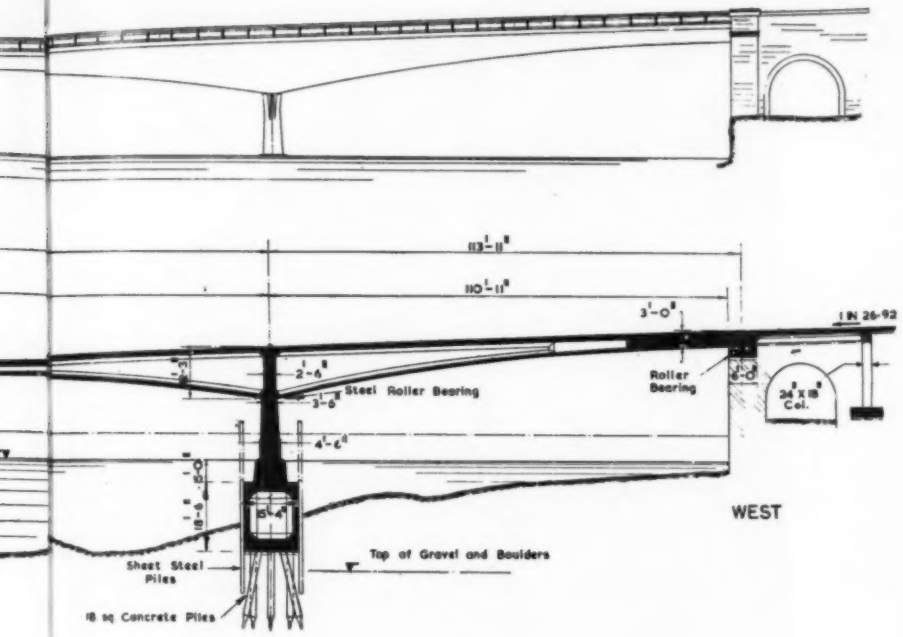
a similar slab forming the carriageway and extending over the top of the four prestressed girders (*Fig.* 1). The slabs forming the footpaths are precast, and the space below will accommodate pipes and cables. The longitudinal girders are continuous over the three spans. Prestressing is by the Freyssinet method.

The two new river piers (*Fig.* 1) are supported on precast concrete piles which were driven about 10 ft. into the gravel under the river-bed and which bear upon a stratum of gravel and boulders. The piers were constructed inside sheet-pile coffer-dams close to two of the old piers (*Fig.* 3). Roller bearings are provided at all supports except the western river-pier at the top of which there is a hinge. The bearings were so placed that after elastic contraction due to prestressing had taken place, the rollers were on the centre-line of their supports.

The new bridge was constructed without any scaffolding in the river. The deck of the old bridge was stripped, leaving only the main longitudinal girders and the transverse beams which were at 12 ft. centres. This framework was then jacked up about 6 ft. off the old piers so that the transverse beams just cleared the deck of the new bridge. The prefabricated plywood shuttering for the new bridge was suspended from the old transverse beams by hangers (*Fig.* 4).

Work began on the two river-piers simultaneously, and continued outwards from the piers in both directions, keeping each part approximately in balance on its pier at all times. The prestressing cables are arranged in layers over the supports, with ends anchored at intermediate construction joints or pockets in upper and lower slabs, so that each part could be prestressed and made self-supporting as it was constructed. The final closure was made at the crown of the middle span. Continuity is effected by a number of cables extending the full length of the bridge and supplemented by comparatively short cables, at the top and bottom, in the middle of the central span.

After the prestressed girder had been



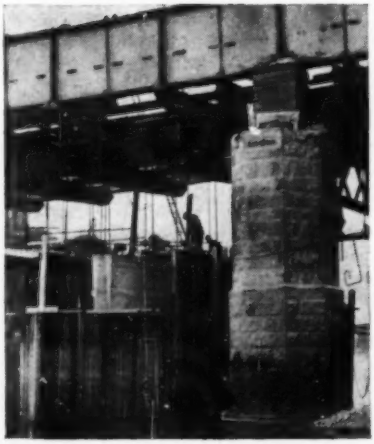


Fig. 3.—Construction of New Pier Adjacent to Old Pier.

completed, the transverse beams of the old bridge were jacked off the new road slab, and separated from the old longitudinal girders which, with their supporting-piers, were then removed. This method enabled the reinforced concrete edge-beams, which are on the line of the old longitudinal beams, to be constructed in shuttering suspended from the transverse girders of the old bridge.

The final stages of the construction were the completion of the service ducts, the removal of the old transverse beams, the surfacing of the roadway, and the laying of the footpath paving slabs.

The scheme for a prestressed bridge of three spans was selected in preference to schemes for strengthening the existing bridge, or for replacing it with a four-span reinforced concrete arch. It was estimated that the bridge would be completed in about fourteen months and would cost about £150,000.

The new bridge is being constructed for the City and Royal Burgh of Perth, the Burgh Surveyor being Mr. J. Penman,

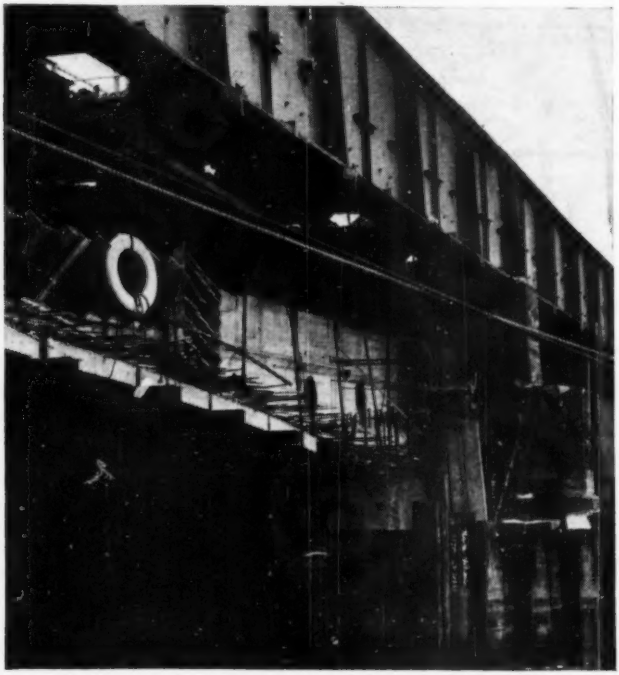


Fig. 4.—Centering for New Bridge Suspended from Old Bridge.

A.M.I.C.E. The consulting engineers are Messrs. F. A. Macdonald & Partners, and the general contractors are Messrs. Whatlings, Ltd.

The following notes on the æsthetics of the old and new bridges are contributed by MR. W. A. FAIRHURST, M.I.Struct.E.

The old Victoria Bridge was erected in 1903. The deck comprised longitudinal and transverse members supporting jack arches. The main longitudinal members were the steel-plate parapet girders which were covered by ornamental cast iron in accordance with the style of the period. The four spans were each about 100 ft. long and there was a fall of about 15 ft. from the east side to the west side of the river. Decorative stonework projected from the piers above the parapet girders and formed features which were popularly known as "cruets". It was not a beautiful bridge and because of the sloping roadway between the parapet girders it was sometimes disrespectfully referred to as the "coal chute".

The new bridge had to be built on the line of the old structure to join up with the main road to Stirling, since any other alignment would require the removal of a number of fine old buildings. The æsthetic problem was not easy to solve and the final solution was to provide three spans for the new bridge. The eastern span is about 30 ft. less than the western span, and the length of the middle span is equal to the sum of the lengths of the end spans. This division of the 390-ft. waterway permitted a maximum depth of deck structure of 8 ft. 6 in. over the pier at the western end and 12 ft. 6 in. over the pier at the eastern end. Because of the double-cantilever type of construction small forces only were transmitted to the abutments, thereby enabling the masonry construction on each bank of the river to be preserved. The variation in depth at the two piers has a balancing effect on the steep gradient of the bridge and the unsymmetrical elevation is an interesting feature. The small depth of

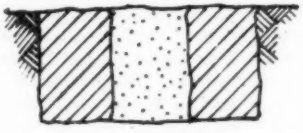
construction at the western pier, that is at the lower end of the bridge, maintains the soffit of the deck structure above the highest known flood level.

Although traffic is now using the bridge, the pavements, services, and approach works are not expected to be completed until September. Her Majesty the Queen has graciously consented to be present at the official opening ceremony which is expected to take place on October 10th, 1960.

Concrete in the Iron Age.

In the year 1937, when on holiday in Herefordshire, the writer came across a small sign marked "To the Roman Camp" (at Poston) and pointing up a rough hillside track. A toilsome trudge led to the site, just under the brow of a hill, where a few members of the Woolhope Naturalists' Field Club were busily at work at the bottom of an excavation. At that time most of the site of the Roman camp had been uncovered and photographed, and the "finds" removed to a museum. The experienced eye of the archaeologist had, however, noticed that the soil below the level of the Roman camp was not an original formation but appeared to be a deposit formed by a silting process during a long period of years. Deeper excavations were therefore made, and under the Roman camp evidence was found of an earlier occupation in which relics of the Early Iron Age were found. The Roman encampment had thus been built on the site of a camp occupied by men in prehistoric times.

The writer soon found himself equipped with tools and was shown where to work.



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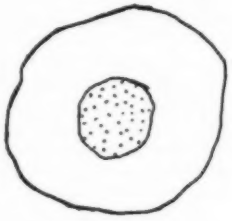


PLAN

He was surprised to find that the tools he had to use resembled table-spoons and forks and that every handful of material had to be searched for anything that was not soil.

The Iron Age habitations were huts formed of small tree trunks for posts with a covering of animal skins or other material. The only indication of the sites of the huts were the holes in which the posts had stood, arranged in rough circles, and many of these had been exposed. As the soil was scraped away at the site of a post a hard substance was encountered, and this was found to surround the hole in which the post had been erected. A completely exposed post-hole was as shown in the sketch, in which the hard material resembling concrete is shown cross-hatched and the space in which the post stood is shown dotted. It is in fact exactly what would be found if the concrete around the bottom of a post erected today were dug out. The space originally occupied by the bottom of the post was filled with rotted wood and silt.

The writer was permitted to take away some pieces of the concrete-like material, and some of these were analysed in the laboratory of the Associated Portland Cement Manufacturers Ltd. The material is speckled brown and white, and the analysis is as follows.

	Brown Portion Per cent.	*White Portion* Per cent.	*Whole Sample* Per cent.
Silica	45·86	27·65	44·92
Alumina	11·46	14·46	11·89
Ferric oxide	3·50	—	3·35
Lime	17·65	22·65	17·56
Magnesia	2·56	1·73	2·50
Carbonic anhydride	11·44	16·56	12·68
Water and loss	6·28	15·94	5·71
Alkalis undetermined	1·25	1·01	1·39
	100·00	100·00	100·00

It is seen that the brown particles are

burnt clay containing some calcium carbonate, and that the white particles are a type of hydraulic lime. It seems reasonable to assume that the two products were burnt separately. The clay would then form a moderately hard aggregate varying in size from dust to small nodules, and the lightly burnt calcareous material would form a lime. When mixed together with the addition of water the two materials would form concrete. The analysis shows that this is not the type of cement used by the Romans.

A guess on the origin of the discovery is as follows. The site of the camp is clay; lumps of calcareous material were imported and rammed into the clay to form hearths for the fires in the middle of the huts; when one of these hearths, after long subjection to fire, was broken up it was found by accident that the material set hard when it was wetted; and this knowledge was applied to the fixing of posts—and perhaps to other uses so far unknown.

H. L. C.

Dynamic Loads on Beams.

In a paper entitled " A Test with Pulsating Load on a Concrete Beam with High-strength Reinforcement ", which was presented at the International Congress of the Precast Concrete Industry held in Stockholm in June last, Mr. Thorsten Lundin (Skanska Cement AB, Sweden) describes tests on a tee-beam having a length of 8 m. (26 ft.), an overall depth of 80 cm. (31 in.), a flange 25 cm. (10 in.) wide, and a web about 10 cm. (4 in.) thick. The beam was tested at an age of one year. The crushing strength of 6-in. cubes at three months was 760 kg. per sq. cm. (10,800 lb. per sq. in.). The tensile reinforcement comprised 8-mm. (0·3-in.) plain bars having a 2 per cent. proof stress of 9 tonnes per sq. cm. (128,000 lb. per sq. in.). The main tensile reinforcement was twenty-two bars placed closely at the centre of the span; towards the ends of the beam the bars were successively bent up into the flange. No stirrups were provided. Under a static load concentrated at four points, the ultimate bending moment of one beam was 83 m.-tonnes (7,200,000 in.-lb.) corresponding to a tensile stress in the reinforcement of 9·6 tonnes per sq. cm. (136,000 lb. per sq. in.).

The pulsating test was carried out with a load applied at the centre of the span. The loading cycle had a frequency of 200 per minute. The load varied from 7 to 15 tons, corresponding to a change of the stress in the reinforcement at the centre of the span from about 1·6 tonnes per sq. cm. (22,750 lb. per sq. in.) to about 2 tonnes per sq. cm. (28,400 lb. per sq. in.), the higher stress corresponding to a total stress of about 4·2 tonnes per sq. cm. (59,700 lb. per sq. in.). There were neither stirrups nor bent-up bars in the middle of the beam for a length of 2 m. (6·6 ft.). At the first loading, a large shearing crack occurred at one side of the loaded point. The width of this crack was 0·7 mm. at the beginning of the test and about 4 mm. at the end (0·03 in. and 0·16 in. respectively). The major enlargement of the crack was caused by the grinding together of the particles of concrete during pulsation. The movements at the crack also caused failure due to bending fatigue of three pairs of horizontal bars in the web and of one bar of the main reinforcement. The concrete strain at a load of 15 tons, and immediately under the load, increased from 0·009 to 0·021 during the pulsation of the load, and the corresponding concrete stress exceeded 400 kg. per sq. cm. (5690 lb. per sq. in.). In other respects the behaviour of the beam was stable.

After a million pulsations the load was increased until shearing failure occurred at a load of 29 tons corresponding to a tensile stress in the reinforcement of 7·4 tonnes per sq. cm. (105,000 lb. per sq. in.), a compressive stress in the concrete of about 720 kg. per sq. cm. (10,240 lb. per sq. in.), and a nominal shearing stress of 24 kg. per sq. cm. (340 lb. per sq. in.).

Prestressed Concrete Hangar at Bombay.

A HANGAR (*Fig.* 1) recently constructed at Bombay for Air India International has two structurally separate parts each with a prestressed barrel-vault shell roof supported by reinforced concrete columns. The main dimensions are shown in *Fig.* 2.

The roof of each half of the hangar has five vaults spanning 150 ft. and supported at the rear by reinforced concrete columns at 14 ft. 10½ in. centres, and at the front by a prestressed concrete beam over the doors. The chord width of the vaults is 29 ft. 7 in. and the thickness of the slab is 2½ in.

The roof beams are 6 ft. 6 in. deep by 7 in. wide and prestressed with eight Freyssinet cables each of twelve 0·276-in. wires. The front beam has a clear span of 160 ft., is 20 ft. deep and is stressed with twenty-seven cables each of twelve 0·276-in. wires.

The hangar was designed for a wind pressure of 20 lb. per square foot and the columns are designed to take the resulting moments together with those created by the elastic shortening of the members when they were stressed. Because of the height of 50 ft., and resultant flexibility, the stresses imposed on the columns by prestressing the beams are not large.

The junction between the two parts of the roof is formed by a reinforced concrete slab fixed to the lower edge of the roof of one half of the hangar and resting on a projection on the edge of the next rib. Between the outermost ribs and the side walls are other reinforced concrete slabs carried by the rib and resting on the side wall-columns. These slabs were separate from the ribs until after the ribs had been prestressed.

The columns supporting the main beams at the front of the hangar are 3 ft. 6 in. square and two separate columns are provided at the junction of the beams. The columns have reinforced concrete bases founded on hard murum, on which the maximum permissible pressure is 6 tons per square foot. Loading tests were made to determine the bearing capacity.

The rear and side walls of the hangar have reinforced concrete frames with brick infilling and vertical glazing set in reinforced concrete glazing bars.

Fig. 1.

Each main front door is composed of five leaves which run on rails at the bottom and are guided at the top by tracks under the canopy attached to the face of the main beam.

The concrete was mixed in the proportions 1 : 2 : 4 with a water-cement ratio between 0·45 and 0·47 giving cube strengths of over 4000 lb. per square inch at seven days. The calculated final stresses in a typical vault are +630 lb. per square inch at the top and +180 lb. per square inch at the bottom, and in the main beam +1050 lb. per square inch at the top and +90 lb. per square inch at the bottom, all of which are compressive and lower than the maximum permitted stress of +1250 lb. per square inch. The stresses are given in *Table* I.

The roof is covered with two layers of aluminium foil which acts as waterproofing and a solar reflector. Each vault is provided with four 5-in. diameter ventilators which help in equalising the pressure inside and outside the building.

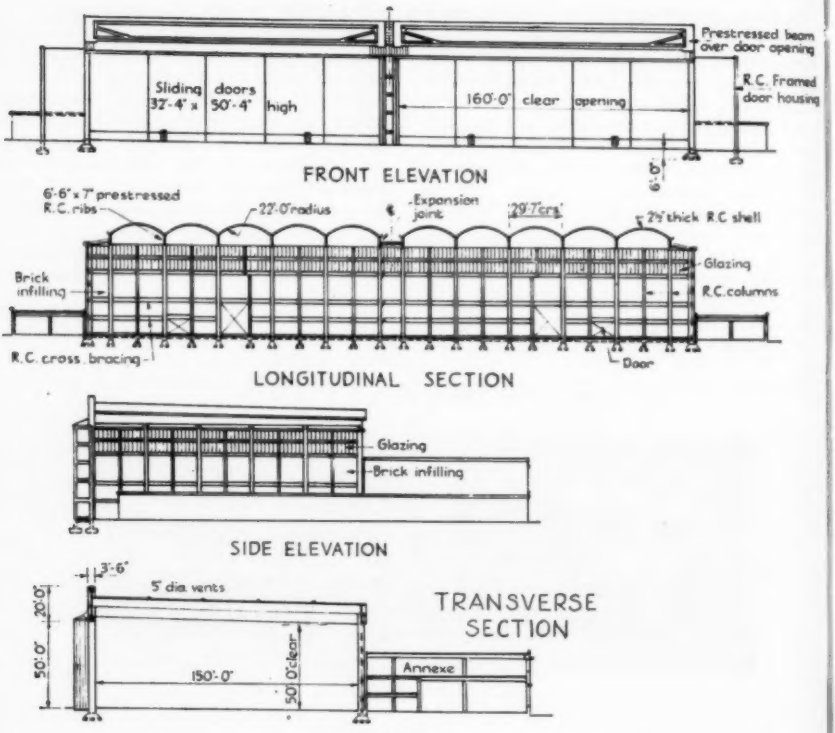


Fig. 2.

TABLE I.—STRESSES (LB. PER SQUARE INCH).

	Typical Vault		Centre-line of Main Beam	
	Top of Vault	Bottom of Vault	Top of Beam	Bottom of Beam
a. Dead weight	+ 890	− 1530	+ 1320	− 1525
b. Prestress (initial)	− 655	+ 2630	− 505	+ 2130
c. Stress at prestressing . . .	+ 235	+ 1100	+ 815	+ 605
d. Prestress after loss	− 550	+ 2220	− 430	+ 1800
e. Full live load	+ 290	− 510	+ 160	− 185
f. Final stress (a + d + e) . .	+ 630	+ 180	+ 1050	+ 90

The concrete surfaces exposed to the weather have been treated with a silicone waterproofing agent to reduce discolouration and absorption of moisture. A two-story building at the rear of the hangar and single-story buildings at each side provide workshops and offices for the maintenance of the aircraft and their equipment.

The consulting engineers were Messrs. Donovan Lee & Partners. The contractors were Messrs. Gammon India Private, Ltd., for the hangar and Messrs. Patkar, Ltd., for the workshops and offices.

A Reservoir with Double Walls.

THE following notes are contributed by MR. R. J. SALTER, B.Sc., A.M.I.C.E., A.M.I.MUN.E.

A rectangular reservoir of 10,000,000 gallons capacity and of unusual construction was built recently for the Department of Public Utilities, St. Louis, Missouri, U.S.A. In many areas of this State there is a high water-table and the water is contaminated with sewage. Therefore underground storage of purified water is not favoured owing to the risk of pollution due to ground-water leaking into the reservoir. To overcome this difficulty the new reservoir has double walls with sufficient space between for easy inspection of the watertightness and for maintenance.

The walls comprise a large number of identical counterforts (*Fig.* 1) more than 30 ft. high and at 16 ft. centres, which support 12-in. reinforced concrete arched slabs on each face. When the reservoir is full the arches are in compression which increases the water-tightness of the concrete.

Because the base of the wall is wide, the pressures imposed on the ground at the edges of the foundation are small. The pressure at the outside of the foundation is about equal to the pressure of the earth filling outside. When the reservoir is empty the unbalanced lateral pressure, due to the earth acting on the outside of the wall, is in part resisted by sheet-piling which is driven below the wall to exclude the ground-water. The lateral pressure of the water in the reservoir is resisted partly by concrete projections into the ground from the underside of the base and partly by the sheet-piling.

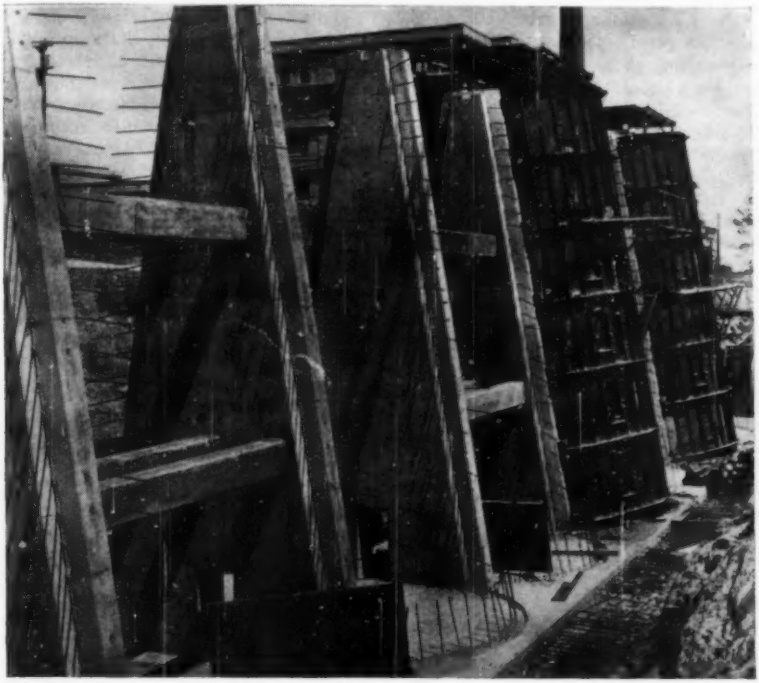


Fig. 1.

There are no expansion or contraction joints in the concrete floor of the reservoir

but a rubber water-stop is incorporated in the joint between the footings of the walls and the floor. The roof of the reservoir is free to move over the walls on all four sides. Movements produced by changes of temperature in the walls are allowed for by bending of the thin arches and by providing beams between the counterforts only in alternate bays as seen in *Fig.* 1.

The roof and floor of the reservoir are level, and both are generally 8½ in. thick with an additional thickness of 4 in. at the columns. The roof is supported by 225 columns which are 30 ft. high and of 20 in. diameter and which were cast in two lifts with an interval of thirty minutes between each lift. The shuttering for the roof was supported by towers of tubular scaffolding, a fifth of the area of the slab being cast at one time. After each part of the roof had acquired sufficient strength, the towers were lowered on jacks and the scaffolding was moved to the next position.

The arched wall slabs were each cast in one lift within metal shutters which were moved from one bay to the next by a crane.

The excavation for the reservoir was made first to within 8 ft. of the required level. The trenches for the wall foundations were then excavated and after the walls had been constructed the remaining 8 ft. of earth was removed from the interior and used as filling outside the walls.

The roof is protected by a layer of felt and bitumen, covered with sand and a layer of earth 4 ft. thick. The inlet pipe of 10 ft. diameter passes through the wall. The two outlet pipes each of 7 ft. diameter pass through the floor and under the foundation.

Mr. E. E. Easterday, of Washington University, who checked the structural design, states that, for walls over 20 ft. high, this form of double-wall construction is cheaper than ordinary counterforted or cantilevered construction.

Steel Girders with Prestressed Concrete Boom.

THE illustration shows a single-lane bridge of four spans at a lumber camp in the state of Washington, U.S.A. The bridge is said to be the first to be built in the U.S.A. with truss girders comprising steel and prestressed concrete. The bridge is 270 ft. long. The two end-spans are of prestressed concrete and each is 35 ft. long. The trusses forming the two central spans, which are each 100 ft. long, are 7 ft. deep. The bottom boom of each truss is of concrete, and is 12 in. by 17 in. in section, and is prestressed with post-tensioned steel. The trusses were fabricated at a precasting works and transported by road. The deck is composed of twenty-six precast concrete slabs. The erection of the trusses and deck was carried out in two days. It is stated in *Engineering News-Record*, January 28, 1960, from which the foregoing is abstracted, that the cost was 12½ per cent. less than that for a bridge entirely of prestressed concrete. (It is unlikely that this relation of costs would apply to a similar structure in Great Britain.) The contractors were Associated Sand & Gravel Co. The consulting engineer was Mr. H. Hadley.

A New Bridge over the River Don.

THE viaduct of seven spans, which is being constructed to carry the Doncaster by-pass over the River Don near Sprotborough, was illustrated in this journal for November 1959.

MR. K. D. WILLIAMS, B.Sc., A.M.I.C.E., of the Bridge Engineer's Office of the West Riding County Council, contributed the notes on the design of the structure given in the following.

At the site of the bridge, the river passes through an alluvial plain about 800 ft. wide and bounded by steep slopes rising to a level of about 100 ft. above the river. The nature of the silt forming the plain precluded the building of an embankment instead of a viaduct, the abutments and piers of which can be founded at a depth of about 40 ft. on a stratum of magnesium limestone underlying the silt. The site is subject to considerable defects due to coal-mining operations, the estimated general subsidence being 10 ft., the differential subsidence being about 10 in., and the stretching and shrinking of the ground being about 1 in. in 100 ft.

Superstructure.

The viaduct will be 760 ft. long and will comprise two separate parallel structures. The level of the deck of the viaduct will be about 70 ft. above the plain and will be on a gradient of 1 in 200 leading to a vertical curve and horizontal transition on the southern part of the structure. By this means excavation and filling for the approaches to the viaduct are more or less balanced. Because of the liability of subsidence, the structure will be statically determinate, expansion joints will be incorporated, and provision will be made for jacking the structure into position after the wave of ground disturbance has passed. The provision of two separate structures enables one carriageway to be in use while the structure carrying the other carriageway is being jacked.

Each structure comprises a central span of 180 ft., four spans of 100 ft. each and two spans of 90 ft. The deck of each of the two structures, between which there will be a gap of 5 ft., will provide a carriageway 24 ft. wide, and marginal strips and verges as shown in *Fig.* 1. The construction is a reinforced concrete deck carried on steel longitudinal girders, and concrete piers and abutments. The deck, which is generally 9 in. thick, is a slab continuous transversely and cantilevering beyond the outer girders. Except for the cantilevers of the river span, which is of double-cantilever and suspended-span construction, and the immediately adjacent spans, the reinforced concrete deck acts as part of the compression flange of the steel girders and thereby a saving of 400 tons of steel is effected.

The bridge is designed to carry the standard Ministry of Transport load Type HA and abnormal load Type HB of 180 tons. An overstress of 15 per cent. will be allowed instead of 25 per cent., as is usual for the abnormal load, to allow for additional stresses resulting from mining subsidence. Allowance is made for a wind of 100 m.p.h. The steelwork is designed in accordance with B.S. No. 153, Parts 3A and 3B. The stresses in the reinforced concrete are in accordance with the Ministry of Transport's Memorandum on Bridge Design and Construction.

The expansion joints are designed to take much larger movements than would be necessary for change of temperature alone. At the end of each 100-ft. span and at the ends of the anchor spans a 2-in. gap between mild-steel angles will be filled with a compressible strip of rubber. The rubber will be compressed under normal conditions in order that it will not be loose when the gap is large, and will be kept in place by means of spikes protruding from the angles. At one end of the suspended span the joint is to be of the "comb" type and fabricated from heavily galvanised mild-steel flats. The joint is to be attached to angle cuts from rolled-steel channels which are to be riveted to the steel girders. The joint will be fully adjustable by slotted bolt-holes and, although it is designed to act as shuttering, for the concrete deck and the asphalt, it will be so made that the metal parts can easily be removed if they require to be replaced.

Slender handrails will be provided. Where the traffic will move at high speed close to the central railings, small flexible guard-rails strong enough to ensure safety are to be fixed. Traffic will be separated from the outer railings by a verge 8 ft. wide which will have a surface of rough asphalt to discourage its use as a part of the highway. The verge will have a

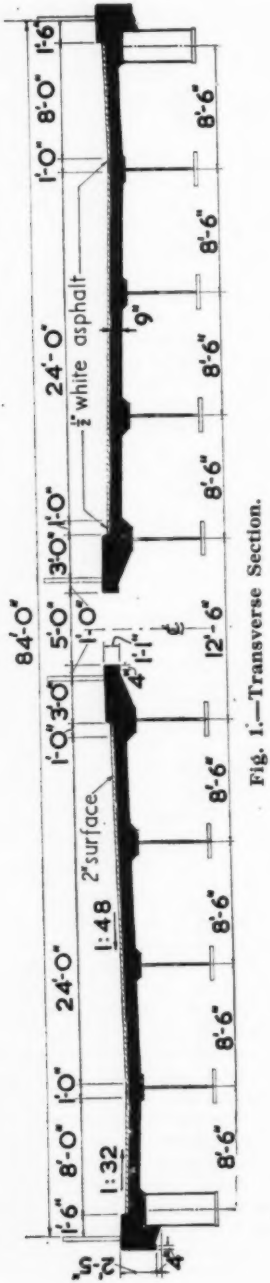


Fig. 1.—Transverse Section.

crossfall of 1 in 30 away from the railings to increase the safety. The road surface will be 2 in. of asphalt on a ¼-in. waterproof membrane. The marginal strips are of white asphalt ¼ in. thick.

Abutments and Piers.

The foundations are in course of construction. The abutments are founded directly on the rock, and are of the counterfort type. They are constructed in four parts, separated by rubber waterstops in order to reduce the effects of mining subsidence. The wing-walls are to be cantilevered to reduce the amount of foundation work.

The piers are in the form of large slender "T's", two of which are placed side by side, one under each carriageway. The columns of the piers are hexagonal in cross-section and are 11 ft. by 9 ft. at the base of the tallest piers and taper to 9 ft. by 7 ft. 6 in. at the top in every case. The columns will be carried on pile-caps which will be below ground. The pile-caps are octagonal in plan except those for the river piers, which are rectangular. There are nine bored piles each of 4 ft. 6½ in. diameter under each octagonal base, and ten similar piles under each rectangular base. There are ninety-four cylindrical piles, the total length of which is 3592 ft. The total amount of concrete in the piles is 1465 cu. yd. Each pile extends into the rock an average distance of 6 ft.; the total length of excavation in rock is 532 ft. The boring equipment is shown in *Fig.* 2. The skip placing the concrete in a tremie for depositing under water in the bores for the piles is shown in *Fig.* 3. Piling operations commenced in July 1959 and were completed in January 1960.

The two northern piers are founded directly on rock. The top of the limestone is soft and weathered, and the piles therefore extend into the rock. It was hoped that the cylindrical linings of the bores for the piles could be dewatered so that the concrete could be placed in the dry, but this method was impracticable and the concrete was deposited under water by means of a tremie. To prevent ingress of silt into the bottom of the pile, the level of water in the cylinder was raised to about 6 ft. above the water-table in the surrounding ground, thereby reversing the flow of water at the cutting edge of the cylinder.

The viaduct is expected to be completed

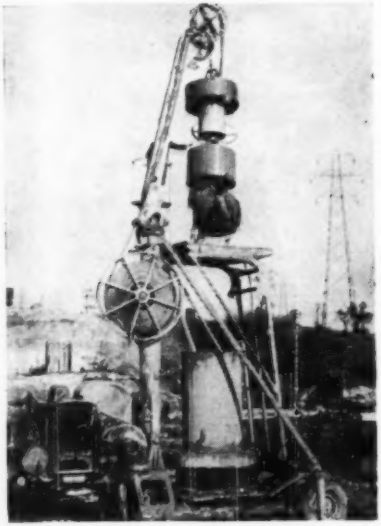


Fig. 2.

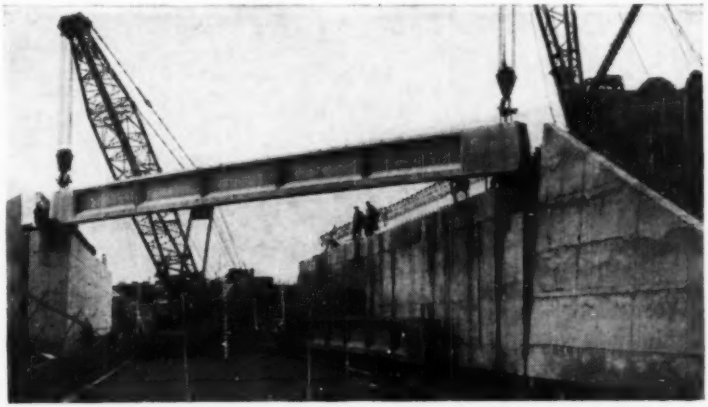
Fig. 3.

during 1961. The cost will be about £650,000. The work has been designed by, and is being carried out on behalf of, the Ministry of Transport under the supervision of the West Riding County Engineer and Surveyor, Colonel S. Maynard Lovell, O.B.E., T.D. The general contractors are Messrs. Cubitts, Fitzpatrick & Shand. The contractors for the piling were Braithwaite Foundations & Construction Co., Ltd.

Precast Beams for a Bridge on the Doncaster By-pass.

THE illustration shows one of eighteen beams each weighing thirty-six tons being placed in position to form the deck of a bridge at Harworth on the Doncaster by-pass road. Each of the prestressed precast beams is 78 ft. long. The cranes are mounted on lorries and one is a 50-ton crane with a 50-ft. jib and the other a 25-ton crane with a 30-ft. jib. The operations of unloading, lifting, and placing in position of each beam occupied an hour; all the beams were erected in three days.

The beams were made by Ferro-Concrete and Stone (North Notts.), Ltd., and were erected by Tarslag, Ltd.



Shell Structures.

THE following particulars relate to recent publications dealing with shell roofs and other structures.

Papers at the Madrid Colloquium.

An International Colloquium on Construction Processes of Shell Structures, organised by the International Committee for Shell Structures in collaboration with the Instituto Tecnico de la Construccion y del Cemento, was held in Madrid in September last. The titles and authors of the papers read at this colloquium, some of which are referred to in the Editorial Note in this number, are given in the following with summaries of some of the papers.

Usine Dunlop à Amiens Sheds en Béton Armé, by M. Hahn (France).

Movable Form Elements, by A. Efsen (Denmark).

Coupole en Béton Armé du Pavillon du Génie Civil a l'Exposition de Bruxelles, by A. Paduart (Belgium).—See this journal for February 1958.

A New Form of Doubly-curved Shell for Roofs and Floors, by G. S. Ramaswamy and Shri S. M. K. Chetty (India).—The casting, testing, and erection of thin slab precast domed slabs for roofs are described. Each slab is 4 ft. square and 1 in. thick and has a peripheral rib. The slab is not reinforced but reinforcement is provided in the ribs. The slabs are supported on precast beams. It is claimed that 50 per cent. less concrete and 40 per cent. less steel are required than in ordinary reinforced concrete construction.

Construction de Voûtes Auto-portantes sur 85,000 m² en Egypte, by J. Barets (Egypt).—The theory of the design of self-supporting barrel-vault roofs with prestressed tie-rods is described and the results of tests to establish the accuracy of the theory are given. The method of construction of roofs covering about 100,000 sq. yd. is described and the cheapness of this form of roof is demonstrated.

Umbrella Type Hyperbolic Paraboloid with Prestressed Edge-Beams, by A. N. S. Kulasinghe (Ceylon).—See this journal for May 1957.

Pannier Market, Plymouth, by A. P. Mason (Great Britain).—See this journal for July 1959.

Construction of a Hyperbolic Paraboloid Shell Roof over the Garage of the Lincolnshire Motor Co., Ltd., Lincoln, by K. Hajnal-Kónyi (Great Britain).—See this journal for January 1960.

Complete Cylindrical Shell Roofs Precast on the Ground, by B. H. Broadbent and R. S. Jenkins (Great Britain).—See this journal for December 1958.

Studies on Models of a Type of Membranal Structure, by Horacio Caminos (U.S.A.).

Shell and Spatial Structure Shapes applied in Poland, by Waclaw Zalewski and Jerzy Krzeminski (Poland).—See this journal for April 1959.

Shell and Folded-Slab Roofs Built with Precast Prestressed Reinforced Concrete Elements, by I. Doganoff, C. Hoffmann, and H. Rühle (Germany).—See this journal for November 1955.

A New Shell Form for Prestressed Sheds, by M. Mihailescu and I. Ungureanu (Roumania).—The form of roof described comprises a number of prestressed thin slabs of S cross section, the centroid of which coincides almost with the centre of the torque. The shape is a cycloidal curve with circular curves at the extremities. Tests to destruction of a full-size element about 39 ft. long, 19 ft. wide, and 2 in. thick were made to confirm the theoretical behaviour of the slabs.

La Conception, l'Exécution et le Comportement d'une coque mince en Béton Précontraint, by H. C. Duyster (Belgium).—The design of a roof in Brussels is described. The form of the roof is two double inverted "umbrellas", each of which comprises four hyperbolic-paraboloidal slabs 2 in. thick. The slabs are prestressed in such a manner that the stresses under dead load and a small live load are uniform throughout. The object was to prevent disturbing stresses due to deformations and, because the structure is so slender, to avoid large deflections. There were, however, some unexpected deflections at the edges of the slabs.

The Construction of Hyperbolic Paraboloid Type Shells without Temporary Formwork, by A. R. Flint and A. E. Low (Great Britain).—The design, construction, merits, and cost of five hyperbolic-paraboloidal thin-slab roofs, at a school in London, are discussed. The slabs

were formed by applying mortar pneumatically from below on to a layer of insulating material supported on a mesh of steel wire. The wires are suspended within permanent steel frames. Some of the wires were post-tensioned, but the concrete was in general prestressed by the weight of the slab upon removal of temporary supports.

Palais des Expositions au Centre National des Industries et des Téchniques à Paris, by N. Esquillan (France).—See this journal for September 1959.

Autres Matériaux: Métaux (Toitures et Parois en Câbles Précontraint), by P. Moenaert (Belgium).

Construction to Third Session, Theme B, by C. Blumfield (Great Britain).

Timber Shell Construction in Great Britain, by H. Tottenham (Great Britain).

Voiles Minces en Céramique Armée; Exemples de Réalisation, by F. Levi and G. Pizzetti (Italy).—Many examples of barrel-vaults and other curved-slab roofs are described of which the novel feature is the provision of hollow-clay tiles.

Theoretical-experimental Research on Circular and Linear Generator Vaults, by Elio Giangreco and Pagano Lei (Italy).

Palais de l'U.N.E.S.C.O. à Paris auvent Piazza, by M. Hahn (France).—See this journal for December, 1959.

Doubly Curved Shells with Straight Line Generators, by J. D. Bennett (Great Britain).—Some aspects of the advantage of rectilinear generators for doubly-curved thin-slab roofs are considered, with special reference to a roof comprising four hyperbolic-paraboloidal slabs, a prestressed roof formed of small precast slabs of the same type, and a roof having the shape of a part of a hyperboloid of revolution. The costs of the structures are discussed.

New Shapes for Shells, by H. Isler (Switzerland).

Centering and Shuttering for Shell Roofs.

Bulletin No. 1 of the International Association of Shell Structures contains the general reports on the meetings of the Colloquium held in Madrid. In particular, details and sketches are given of various methods adopted for the construction, the centering and shuttering for shell roofs.

Model Shell Structures.

The following paper was presented at the annual Convention of the American Concrete Institute.

"Experiments with Thin-Shell Structural Models." By J. L. Waling.—A new method of forming hyperbolic-paraboloid shell structures depends upon the geometrical characteristics of such structures and makes use of lightweight concrete supported on wires acting as permanent shuttering. Experiments of the proposed construction made use of space models and construction models.

Descriptions of models of some shell structures are given in the number of this journal for July 1960, namely, hyperbolic-paraboloidal roofs at Lincoln (page 284) and at Bedford (page 285), and a large dome of precast construction at Belgrade (page 265).

Colloquiums on Shell Structures.

The International Association for Shell Structures is to hold a colloquium, in conjunction with the International Union of Testing and Research Laboratories for Materials and Structures (RILEM), on research on shell structures. This colloquium will he held in Delft, Holland, from August 30 to September 2, 1961. Professor Haag of Delft Techische Hogeschoo is associated with this meeting. Particulars can be obtained from Professor A. L. Bouma, Technological University, Delft.

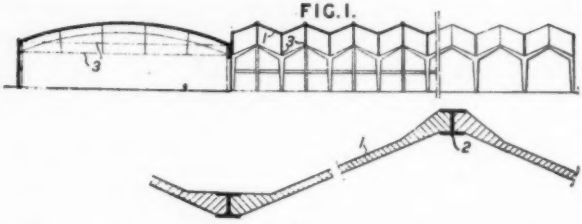
Another colloquium dealing with simplified methods of calculation of shell structures is to be held by the Association in Brussels from September 4 to September 6, 1961. Professor A. Paduart is associated with this meeting. Particulars can be obtained from Professor Dutron, 127 Av. Adolphe Buyl, Brussels 5.

The colloquium on precast shells, a preliminary announcement of which appeared in this journal for February 1960, is to be held in Dresden in the early part of November 1960. Dr. Rühle, in conjunction with the Kammer der Technik, is associated with this meeting, particulars of which can be obtained from Dr.-Ing. H. Rühle, Dostojewskistrasse 10, Dresden-Loschwitz, D.D.R.

Patent Applications for Shell Roofs.

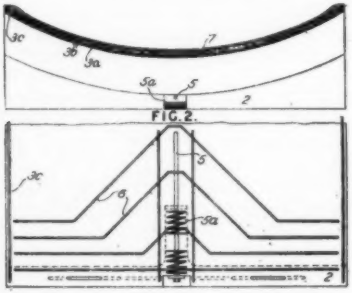
A concrete roof (*Fig.* 1) comprises prefabricated concrete shells 1 curved to span the building transversely and sloping in opposite directions in the longitudinal direction of the building, the adjoining edges of the shells providing mutual sup-

FIG. 1.



port between the shells at the ridges and valleys formed by the shells. The adjoining edges of the shells may directly abut or they may be thickened and supported by curved I-section steel girders 2 which may be strengthened by tie rods 3 and grouted in position. The shells may be provided with prestressing cables running inside tubes built into the shells.—No. 828,467. W. J. Silberkuhl. April 17, 1957.

FIG. 2.



A concrete roof (*Fig.* 2) comprises a section of a hyperboloid, is of rectangular shape in plan, is supported at its opposite ends 2, and has one or more ties 5 extending between the supporting ends which transmit the stresses of the shell. The side edges of the shell are formed free from projecting webs. The shell reinforcement 3*a*, 3*b* runs with its main supporting direction at right angles to the tie rod 5, and has distributing bars 3*c* in the longitudinal direction of the shell. The tie rod 5 spans the underside of the shell, being anchored at 5*a* in the ends 2 and emerging below the shell where it leaves the anchorages, the forces within the tie rod 5 being distributed to the shell by bars 6. In modifications, (diagrams not shown), the tie rods run diagonally across a shell but are embedded therein.—No. 827,933. W. J. Silberkuhl. October 4, 1957.

A British Standard Code for Rendering.

THE NEW British Standard Code of Practice* describing current practice relating to external rendered finishes for protective and decorative purposes is limited to the consideration of ordinary cement-lime-sand mortars applied on most types of old or new backgrounds. The types of renderings include trowelled or floated plain surfaces, scraped or textured finishes applied by hand, rough-cast thrown wet by hand, dry-dash thrown by hand, and renderings applied by machines, which may be operated by hand or power. The importance of having a satisfactory background is stressed, particularly its strength and porosity. The necessity of taking precautions to offset the detrimental effects of soluble salts in the background and the penetration of water is dealt with. Backgrounds such as concrete blocks, dense concrete, lightweight concrete, and no-fines concrete are considered. Durability in relation to crazing, cracking and atmospheric pollution is also considered. The mixtures recommended for certain types of finish applied to various types of background and conditions of exposure are tabulated. The maintenance and repair of existing rendered finishes, including stucco, are also dealt with, but the Code does not deal with rendering for the retention of liquids.

* British Standard Code of Practice CP.221 (1960): "External Rendered Finishes". Price 8*s.* 6*d.* (plus postage) from the British Standards Institution, 2 Park Street, London, W.1.

Concreting in Cold Weather.

Two publications on concreting in cold weather have been issued by the Finnish State Institute for Technical Research, Helsinki. No prices are stated.

"Talvibetonointi Työmaatutkimus" (Winter Concreting), by Arvo Nykänen, is printed in the Finnish language with a short summary in English. An account is given of a study of the construction of reinforced concrete framed structures during a period of four years in Helsinki, and the methods of preventing the freezing of the concrete are described. The costs of the protective measures as a proportion of the total cost of the structure are given; these ranged from 6 per cent. to 22 per cent. for removing ice and snow from the shuttering, reinforcement, and elsewhere; from 7 per cent. to 36 per cent. for heating the materials and water before they were mixed; and from 2 per cent. to 14 per cent. for protecting the concrete while it was hardening.

"Koetalo Talvibetonointitutkimuksen Apuna", by Arvo Nykänen and S. E. Pihlajavaara (printed in the Finnish and English languages), is a report of observations made during the concreting of a structure which was built specially for the purpose of studying the efficiency of various methods of protecting concrete against the effects of low temperatures. Descriptions and illustrations are given of the methods of construction and protection and of the instruments used, and there is a discussion on the application of the data.

Other publications received recently are as follows.

"Winter Concreting: Research carried out on Building Sites." By A. Nykänen. (Helsinki: State Institute for Technical Research, Finland. Publication No. 45. 1959. In Finnish with English summary.)

"The Experimental House as an Aid in Winter Concreting Research." By A. Nykänen and S. E. Pihlajavaara. (Helsinki: State Institute for Technical Research, Finland. Publication No. 44. 1959. In Finnish and in English.)

Building Trades Exhibition.

THE twenty-third Building Trades Exhibition is to be held in the City Hall, Manchester, from October 11 to 22, 1960. Particulars can be obtained from Mr. T. Phillips, 1 Brazennose Street, Manchester, 2.

Bulletins Received.

"**Soil Mechanics Aspects of the Design of Structures in Areas of Mining Subsidence.**" By J. D. Geddes. (Bulletin No. 19 of the Department of Civil Engineering, University of Durham. Price 2*s.* 6*d.*)

"**A Study of the Granulators used in the Production of Roadmaking Aggregates.**" Road Research Technical Paper No. 44. (London: H.M.S.O. 1960. Price 5*s.* 6*d.*)

"**Allmän Teori för Beräkuing av Amerad Betong.**" By H. Granholm. (Gottenburg: Transactions of Chalmers University of Technology. No. 209. 1959. In Swedish with summary in English. Price kr. 20.)

"**Remarks about the Design of Reinforced Concrete Beams.**" By J. Ferry Borges and J. Agra E. Lima. (Lisbon: Laboratorio Nacional de Eugenharia Civil. Technical Paper No. 122. 1958. In English.)

Reinforced Concrete Association.

THE President of the Reinforced Concrete Association for the 1960 session is Mr. W. Kirby Laing, M.A., M.I.C.E. (John Laing & Son, Ltd.). The Vice-president is Mr. A. P. Mason, B.Sc., M.I.C.E., M.I.Struct.E. (British Reinforced Concrete Engineering Co., Ltd.). Newly elected ordinary members are Mr. F. G. Clarke, M.I.Struct.E. (Messrs. Clarke, Nicholls & Marcel), Mr. H. G. Cousins, B.Sc., M.I.C.E., M.I. Struct.E., Mr. K. H. Brittain, A.M.I.C.E., A.M.I.Struct.E. (Associated Portland Cement Manufacturers, Ltd.), Mr. M. G. Cowlishaw [Guest, Keen & Nettlefolds (South Wales), Ltd.], and Mr. D. H. New, E.R.D., B.Sc., M.I.C.E., M.I.Mech.E., M.I.Struct.E. [Holland & Hannen and Cubitts (Great Britain) Ltd.].

Constructional Plant Exhibition.

AN exhibition of constructional plant to be held at the site of the Crystal Palace, London, in June 1961, is being organised on behalf of the industry by Construction Equipment Exhibitions Ltd., of Drury House, Russell Street, London, W.C.2, from which company particulars are obtainable. The exhibition is the successor to exhibitions of building and civil engineering plant held recently by the Ministry of Works.

A Flame-tool for Cutting Concrete.

A FLAME-CUTTING tool was used recently in the U.S.A. to burn through reinforced concrete walls 18 in. thick and 17 ft. high (*Figs.* 1 and 2) at a rate of 2 to 3 ft. per hour, and to pierce the walls in about 2½ minutes. The tool operates by the burning of oxygen and a metallic powder at the end of an iron tube which is fitted into the tool and is consumed during the operation. The tube is gripped in a chuck on the tool which is connected by flexible tubes to the oxygen and powder supplies, which are mixed as they pass along the tube and burn at the end at a temperature of about 8000 deg. F. For concrete, a mixture of iron and aluminium powders is used which increases the temperature of the flame. The materials required to cut 1 ft. of concrete 1 ft. thick are: 1000 cu. ft. of oxygen, 40 lb. of metallic powder, and 10 ft. of black-iron pipe. It is claimed that there is no practical limit to the thickness of the material that can be pierced by this means and that little noise or vibration is produced.

The tool is called the Oxweld ACL-4 Powder Lance and the metallic powders are obtainable from the Union Carbide International Company, New York.

Fig. 1.—Cutting Concrete.

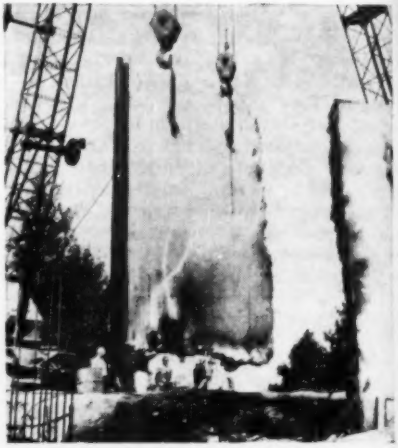


Fig. 2.—Removing a Section Cut from a Wall.

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